APPENDIX B
EROSION ANALYSIS

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BASIS OF DESIGN REPORT
JORGENSEN FORGE EARLY ACTION AREA

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LIST OF ACRONYMS AND ABBREVIATIONS

ACES	Automated Coastal Engineering System
AOC	Administrative Order on Consent
CERCLA	Comprehensive Environmental Response, Compensation, and Liability Act
cf/s	cubic feet per second
cm/s	centimeter per second
EAA	Early Action Area
Em	Engineering Manual
EMJ	Earle M. Jorgensen Company
EPA	U.S. Environmental Protection Agency
FEMA	Federal Emergency Management Agency
FIS	Flood Insurance Study
ft/s	feet per second
H:V	horizontal to vertical
Jorgensen Forge	Jorgensen Forge Corporation
LDW	Lower Duwamish Waterway
MLLW	mean lower low water
mm	millimeter
MSL	mean sea level
NAVD88	North American Vertical Datum of 1988
NGVD	National Geodetic Vertical Datum
NOAA	National Oceanic and Atmospheric Administration
NTCRA	non-time-critical removal action
Owner	EMJ and Jorgensen Forge
psi	Pounds per square inch
psf	Pounds per square foot
RAB	removal action boundary
RM	river mile
Site	Jorgensen Forge project site
SOW	Statement of Work

STAR	Sediment Transport Analysis Report
STM	Sediment Transport Modeling Report
USACE	United States Army Corps of Engineers
USGS	United States Geological Survey
WSDOT	Washington State Department of Transportation

1 INTRODUCTION

This Erosion Analysis Report was prepared on behalf of Earle M. Jorgensen Company (EMJ) and Jorgensen Forge Corporation (Jorgensen Forge; herein referred to collectively as the Owner) pursuant to the Administrative Settlement Agreement and Order on Consent for Removal Action Implementation (AOC; U.S. Environmental Protection Agency [EPA] Region X Comprehensive Environmental Response, Compensation, and Liability Act [CERCLA] Docket No. 10-2012-0032) and attached Statement of Work (SOW). This Erosion Analysis Report is an appendix to the Basis of Design Report (BODR) Final Design submittal for the cleanup of contaminated sediments and associated bank soils in a portion of the Lower Duwamish Waterway (LDW) Superfund Site adjacent to the Jorgensen Forge facility (Facility) located in Tukwila, King County, Washington (see Figure 1 of the BODR; Jorgensen Forge Early Action Area [EAA]). The cleanup will be conducted as a non-time-critical removal action (NTCRA) in accordance with EPA's selected cleanup alternative documented in the *Action Memorandum for a Non-Time-Critical Removal Action at the Jorgensen Forge Early Action Area of the Lower Duwamish Waterway Superfund Site in Seattle, Washington* (Action Memo; EPA 2011) and detailed in the *Final Engineering Evaluation/Cost Analysis [EE/CA]– Jorgensen Forge Facility, 8531 East Marginal Way South, Seattle, Washington* (Anchor QEA 2011). The Jorgensen Forge EAA is located near River Miles (RMs) 3.6 to 3.7 on the east bank of the LDW.

The limits of the Jorgensen Forge EAA (herein referred to as the removal action boundary [RAB]) extend from the top of the bank at approximately 19 to 20 feet mean lower low water (MLLW) (or top of the sheetpile/concrete panel on the southern portion of the Facility) to the federal navigation channel. The RAB is bounded to the north by The Boeing Company Plant 2 Duwamish Sediment Other Area and Southwest Bank Corrective Measure EAA cleanup area, as specified in the EPA-approved Memorandum of Understanding (EMJ et al. 2007). EPA identified this cleanup area as the northern portion of the Jorgensen Forge EAA.

As detailed in the BODR, the EPA-approved removal action alternative selected for the Jorgensen Forge EAA involves the removal of impacted sediments and placement of backfill material within the RAB. The purpose for the backfill material placement is to return the mudline to approximately the original grade. The shoreline bank within the RAB will also be reconfigured and stabilized to contain the underlying soils and minimize the potential for erosion.

The potential for erosion of the in-water backfill material and armoring layers on the shoreline bank depends on the following erosive processes that are likely to occur in the LDW:

- Waves generated by passing vessels
- Localized propeller wash from vessels
- Currents in the river

Each of the potential erosion forces identified was evaluated independently to determine the design requirements for the backfill and shoreline bank armoring protection being required. Based on a review of the LDW shoreline geometry in the RAB, significant wind-generated waves are not expected due to the limited fetch distances.

This appendix details the evaluation of the potential erosive forces on the shoreline and on the backfill. Recommendations on grain size are made from an analysis of these erosive forces.

The appendix is organized as follows:

- Section 2 presents the vessel wake analysis
- Section 3 presents the propeller wash analysis
- Section 4 presents the current analysis
- Section 5 presents a summary of the recommended materials

2 VESSEL WAKE EVALUATION

This section describes the analysis used to determine the stable particle sizes along the shoreline necessary to resist vessel-generated wakes from recreational and commercial vessels that operate near the RAB.

Vessel-generated waves from vessels transiting along this reach of the LDW were computed using the methods presented in Sorensen (1997). The selection of the design vessels identified and the operating criteria stated previously was based on previous studies conducted around the RAB, which evaluated vessel traffic within this specific reach of the LDW, primarily AMEC (2011) and LDW Group Sediment Transport Analysis Report (Windward and QEA 2008). The following vessels were selected for the evaluation:

- Island Tug and Barge Company Tugboat *Patricia S*
- Foss Maritime Tugboat *Wedell Foss*
- Olympic Tug and Barge *Tugboat J.T. Quigg*
- Manson Construction *Derrick Barge* #24
- Recreational Vessels: Yachts varying in length range from 100 to 160 feet

The physical characteristics of these vessels are listed in Table 1.

Table 1
Summary of Design Vessel Characteristics

Vessel	Vessel Length (feet)	Vessel Beam (feet)	Vessel Draft (feet)	Vessel Displacement (lbs)
Tug Boat Patricia S	92	25	9	20,700
Tug Boat Wendell Foss	93.6	36	16.5	55,598
Tug Boat J.T. Quigg	98	29.5	12.3	35,559
Barge Mason Derrick #24	200	84	6	100,800
Yacht - 100-foot	100	24	8	19,200
Yacht - 160-foot (Trinity Euphoria 168 feet)	168	28	7.6	35,750

Note:
lbs = pounds

The design criteria selected for the vessel wake analysis is as follows:

- Design high water level (mean higher high water [MHHW]) = 11 feet MLLW Vertical Datum
- Design low water level (MLLW) = -5 feet MLLW
- Federal navigation channel depth = -15 feet MLLW
- Distance from sailing line to edge of shoreline = 100 feet
- Vessel design speed = 7 knots (posted speed limit per AMEC and Floyd Snider 2011)

The predicted vessel-generated wave heights expected to be generated near the Site ranged between 0.5 feet to 1.5 feet with periods of approximately 2 seconds, as shown in Table 2. These results were compared to similar analysis conducted for sites near the RAB (AMEC and Floyd Snider 2011; Windward and QEA 2008) and are consistent with those findings.

An example of the vessel wake calculations for the tugboat *Patricia S* (including design parameters and assumptions) are included as Attachment A. The results are summarized in Table 2 as follows.

Table 2
Summary of Design Vessel-Generated Wakes

Vessel	Water Depth (feet)	Vessel-Generated Wave Height (feet)	Vessel-Generated Wave Period (s)
Tug Boat *Patricia S*	15	1.4	1.9
	26	1.4	1.9
Tug Boat *Wendell Foss*	--	-	-
	26	1.0	1.9
Tug Boat *J.T. Quigg*	15	1.3	1.9
	26	1.2	1.9
Barge *Mason Derrick #24*	15	0.7	1.9
	26	0.6	1.9
Yacht – 100-foot	15	0.5	1.9
	26	0.5	1.9
Yacht - 160-foot (*Trinity Euphoria* 168-foot)	15	0.4	1.9
	26	0.4	1.9

Note:
The *Wendell Foss* cannot travel past the RAB at low tide due to draft limitations.
s = seconds

The Automated Coastal Engineering System (ACES) Rubble Mound Revetment Design Module was used to compute the armor stone gradation and thickness required on the shoreline to protect against the 1.5-foot, 2-second vessel-generated wake. ACES is a computer program developed by the U.S. Army Corps of Engineers (USACE) in 1992 and is an accepted worldwide reference for modeling water wave mechanics and properties and sizing coastal design structure (USACE 1992). This particular design module of ACES assumes that the waves would propagate and break on the slope of the armor layer.

The computed stable stone size and required filter layer for a restored shoreline slope of 2 Horizontal: 1 Vertical (2H:1V) results in a stable stone size of 0.4 feet and 0.05 feet, respectively, as presented in Table 3. The armor stone calculations (including design parameters and assumptions) are included as Attachment A to this appendix.

Table 3
Summary of Shoreline Armor Stone and Filter Layer Sizing for Vessel-Generated Wakes

Armor Layer			Filter Layer		
Thickness percent less than by weight	**Weight (lb)**	**Dimension (feet)**	**Thickness (percent less than by weight)**	**Weight (lb)**	**Dimension (feet)**
0 (minimum)	1.3	0.2	0 (minimum)	0.00	0.03
15	4.1	0.3	15	0.01	0.03
50	10.2	0.4	50	0.02	0.05
85	20.0	0.5	85	0.06	0.07
100 (maximum)	40.8	0.6	100 (maximum)	0.11	0.09
Thickness = 0.8 feet			Thickness = 1.0 feet		

Notes:
lb = pound

3 PROPELLER WASH EVALUATION

This section describes the analysis used to determine the stable particle sizes for the backfill material necessary to withstand the erosive forces associated with propeller wash from commercial vessels that operate within the federal navigation channel adjacent to the RAB.

The propeller wash analysis was conducted based upon the methods presented in EPA's "Armor Layer Design for the Guidance for In-Situ Subaqueous Capping of Contaminated Sediment" (Maynord 1998). The Maynord method is based on the relationships developed by Blaauw and van de Kaa (1978) and Verhey (1983). This method considers the physical vessel characteristics (e.g., propeller diameter, depth of propeller shaft, and total engine horsepower) and the operating/site conditions (e.g., applied horsepower and water depth) to estimate the propeller-induced bottom velocities at various distances behind the propeller.

Using the transient vessels identified as frequenting this reach of the LDW during the vessel-generated wake analysis (Section 2), the propeller wash analysis was conducted using the vessel characteristics for the vessel with the highest bottom velocity operating over the backfill placement. In this analysis, the tugboat *Patricia S* was identified as the design vessel.

The vessel characteristics in Table 1 were used in this analysis with the following additional design criteria:

- The vessel would be maneuvering directly over the placed backfill material in the RAB
- Distance to the propeller shaft to the channel bottom = 7 feet at design low water (MLLW= 0 feet); 18 feet at design high water (MHHW = 11 feet)
- Engine horsepower = twin 2400 horsepower engines
- Applied engine horsepower = operating at 25 percent applied power during maneuvering conditions
- Propeller diameter = 6.3 feet
- Propeller system = ducted propeller

This analysis indicates that the efflux jet velocity exiting the propeller is 15.1 feet per second (ft/s).

The distribution of jet velocity from the propellers was also analyzed to determine the propeller wash velocity at the toe of shoreline slope. The methods of Blaauw and van de Kaa (1978) were used for this analysis purpose. The results indicate that at a distance of 60 feet, the jet velocity acting on the toe of the shoreline slope is negligible.

To determine the stable sediment size required to protect against the jet velocity, the guidance presented in Chapter 3 of the EM 1110-2-1601 manual (USACE 1994) and modified by Maynord was used. The modified approach, which using a gradation factor and bases stone size on D_{50}, relates velocity to stone size (Palermo et al. 1998).

The predicted 15.1 ft/s design jet velocity results in a stable rock size with a D_{50} of 0.9 feet when the vessel is operating during low water and a D_{50} of 0.1 feet when the tugboat is operating during a high tide.

Propeller wash calculations for low water and high water operations for the *Patricia S* (including design parameters and assumptions) are included as Attachment B.

Additionally, the estimates of potential surface sediment mixing and scour depths due to propwash forces based on the design vessels and operating parameters for coarse grain sand backfill material was determined. It is worth noting that “mixing” and “scour” are related concepts and the extent to which a particular force of the sediment will cause mixing of existing sediments or scour and movement of those sediments to another location has primarily to do with other aspects of the long-term hydrodynamic and sedimentation regime present in any particular area. For example, mixing may be the predominant outcome of propwash forces in relatively quiescent areas where the disturbed sediment essentially falls back to the sediment bed at or near its previous location. Likewise, a dynamic balance or equilibrium of these conditions may exist over time in some areas. To recognize these more complex aspects of propwash effects on surface sediment, the term “disturbance” is used here.

The Hamill (1988) method was used to predict the disturbance and mixing depth. This method is based on the clearance of the propeller tip above the bed, the diameter of the propeller, jet velocity at the bed, sediment grain size, and time of exposure to the propeller wash (a time rate of scour). For this method, an exposure time of 120 seconds (2 minutes) and 300 seconds (5 minutes) was used.

For the design vessel, tugboat *Patricia S*, the results indicate that for a coarse grain sand (D_{50} = 0.2 millimeters [mm]) at an exposure time of 120 seconds, there is a 1.5-inch scour potential and at an exposure time of 300 seconds, the scour potential is 1.9 inches. Therefore, the use of a coarse sand with the proposed backfill placement thickness ranging from 1 to 9.5 feet would provide a sufficient residual mixing layer and meet the substrate conditions of the existing riverbed.

4 DETERMINATION OF RIVER CURRENTS

The following section summarizes the hydrodynamic and sediment transport characteristics of the LDW in the vicinity of the RAB (approximately RMs 3.6 to 3.7; where RM 0 is at the confluence of the East and West Waterways) during a 100-year return period event. This study was based on information provided in the King County, Washington Flood Insurance Study (FIS) (FEMA 2005) and the Sediment Transport Analysis Report (STAR) (QEA and Windward 2008) and Sediment Transport Modeling Report (STM) (QEA 2008). No additional modeling has been completed for this evaluation.

4.1 High-flow Events in the Green River

Table 4 shows the flow rates for the 2-, 10-, and 100-year return period events for the Green River (taken from Table 3-1 of the STM). The 100-year flow is 12,000 cubic feet per second (cfs).

Table 4
High-flow events in the Green River

Return Period (year)	Peak Flow Rate (cfs)
2	8,400
10	10,800
100	12,000

Note:
cfs = cubic feet per second

4.2 Water Surface Elevations

The 100-year base flood elevation at RM 3.6 is 8.5 feet National Geodetic Vertical Datum (NGVD) 29 (Panel 19P of the FEMA FIS for King County, Washington). This corresponds to 12.0 feet North American Vertical Datum of 1988 (NAVD88) and 14.3 feet MLLW based on tidal datum information from National Oceanic and Atmospheric Administration (NOAA) Station 9447130 at Seattle, Washington. Other return periods are not shown; however, because the magnitudes of the 2-year and 10-year flow rates are 70 percent and 90 percent, respectively, of the magnitude of the 100-year flow rate; given similar tidal conditions, the lower return period events likely have a similar base flood elevation.

4.3 Bottom Shear Stress and Stable Grain Sizes in the Channel (Off-Slope)

Calculated maximum bottom shear stresses due to skin friction from the LDW model are found in STM Figures E-6 through E-8 for the 2-, 10-, and 100-year return period flows. The maximum of the range of values was extracted from the model cells between RM 3.6 and 3.7. The corresponding cell depths were taken from STM Figure 2-3 (the modified grid). The shear stress/grain size relationships presented in Table 7 of Scientific Investigations Report 2008–5093 (USGS 2008) was used to relate shear stress to stable grain size. Table 5 presents these values for the model cells in rows 2 and 3 (in the upstream/downstream direction) from the eastern boundary of the grid between RM 3.6 and 3.7. As a conservative measure, grain sizes were rounded up to the next millimeter.

Table 5
Bottom elevation, maximum bed shear, and calculated stable grain sizes adjacent to site

Event Return Period (year)	Range of bottom elevations (feet MLLW)	Maximum bed shear stress (psf)	Stable grain size (mm)
2	-3.4 to -8.4	0.04	3.0
10	-3.4 to -8.4	0.06	5.0
100	-3.4 to -8.4	0.06	5.0
100	-13.4 to -18.4	0.08	6.0

Notes:
psf = pounds per square foot
mm = millimeter
MLLW = mean lower low water

Bed shear stresses and stable grain sizes provided in Table 5 are applicable to all areas (in channel and off-slope) within the extent of proposed dredging and backfilling. The resolution of the existing hydrodynamic model in the project vicinity is too coarse to develop separate design criteria for each sub-area. Therefore, the largest bed shear stress predicted by the existing model in the RAB was used to develop the stable grain sizes listed in Table 5.

4.4 Bottom Velocities Adjacent to RAB

Figure E-22 of the STM shows the near-bottom velocities and shear stresses during a 100-year flood event at a model cell close to shore in the vicinity of RM 3.5 (see STM Figure E-18). The velocity peaks at around 2.8 to 3.0 ft/s, while bed shear shows a maximum value of about 0.07 psf.

To validate the velocities taken directly from STM Figure E-18, another method to estimate velocities near the RAB was employed. This entails calculating the velocity magnitude from the shear stresses presented in Table 5, using Equations A-2 through A-4 in Appendix A of the STM. Details of the calculation are shown in Attachment C.

Table B-6 of the STM shows that the D_{90} of the cohesive sediment on the eastern bench near the RAB is 940 microns. From Table 2, the range of bottom elevations is -3.4 to -18.4 feet MLLW, and Figure E-4 of the STM shows the range of water surface elevations to be from -2.36 feet to 12.64 feet MLLW. Given the potential range of water depths from 1 foot to 31 feet, there is significant variation in the resulting calculations of bottom velocity based on the given shear stresses. Table 6 shows the results of the velocity calculations, which range from 1.6 ft/s to 4.1 ft/s; encompassing the values shown in STM Figure 3-18.

In summary, the maximum bottom velocity in the vicinity of the RAB ranges from 2.8 ft/s for the 2-year event to 4.1 ft/s for the 100-year event. These values are within the range of velocities that were read from STM Figure E-18, and provides an additional line of evidence for estimating near bed velocities adjacent to the RAB.

Table 6
Calculated Bottom Velocities (from Predictions of Maximum Bed Shear) Adjacent to RAB

Event Return Period (year)	Maximumbed shear stress (psf)	Bottom elevation (feet MLLW)	Bottom Calculated Velocity (ft/s)
2	0.04	-3.4	1.6
		-8.4	2.8
10	0.06	-3.4	2.0
		-8.4	3.4
100	0.06	-3.4	2.0
		-8.4	3.4
100	0.08	-13.4	3.6
		-18.4	4.1

Notes:
psf = pounds per square foot
ft/s = feet per second
MLLW = mean lower low water

4.5 Estimated Depth-averaged Velocities

To determine the necessary riprap stone size to prevent erosion of the shoreline bank, the depth-averaged velocity in the federal navigation channel must be determined. The velocities calculated in the previous section represent near-bottom current velocities which were used to evaluate stable grain sizes for in-channel, off-slope areas. For sizing armor along the shoreline bank, a depth-averaged velocity over the entire water column is more appropriate. The depth-averaged velocity can be estimated using the "law of the wall", in which an idealized logarithmic velocity is developed from the bottom shear stress and the flow and sediment characteristics (Dingman 2009); calculations are detailed in Attachment C.

As before, D_{90} is equal to 940 microns. Velocity profiles were calculated for two 100-year scenarios; one with a total water depth of 21 feet and a bottom shear stress of 0.06 psf, and the second for a depth of 31 feet and a bottom shear of 0.08 psf. These conditions correspond to two separate model cells adjacent to the RAB as taken from the STM. Table 7 presents the depth-averaged velocities calculated using this method. The maximum depth-averaged velocity for the 100-year event 5.7 ft/s.

Table 7
Depth-Averaged Velocities Calculated Using the 'Law Of The Wall'

Event Return Period (year)	Maximum bed shear stress (psi)	Total water depth (feet)	Near-bottom velocity (ft/s)	Depth-averaged Velocity (ft/s)
100	0.06	21	3.4	4.7
100	0.08	31	4.1	5.7

Note:
ft/s = feet per second
psi = pounds per square inch

4.6 Riprap Stone Size Calculations

The Maynord formulation (Appendix A of Palermo et al. 1998) was used to estimate a median riprap diameter and weight based on the largest 100-year depth-averaged velocity in Table 7 of 5.7 ft/s. The formula is detailed in Attachment C. Since site-specific hydrodynamic modeling was not performed in support of this evaluation, there is some uncertainty in the magnitude of the 100-year depth-averaged current velocity in the LDW adjacent to the RAB. Therefore, a safety factor of 2.0 was applied to calculations of stone sizing for the banks.

Stone size calculations were carried out for a shoreline slope of 2H:1V and depths from 0.1 feet to 30 feet. Table 8 summarizes the results of the analysis at a depth of 1 foot.

Table 8
Median armor weights and diameter for shoreline slope at a depth of 1 foot

Shoreline Slope (H:V)	Median Diameter[1] (feet)	Median Weight[1] (lb)
2H:1V	0.9	114

Note:
1 = Safety factor of 2 applied to 100-year depth averaged velocities

The Washington State Department of Transportation (WSDOT) material gradation best meeting the shoreline armor layer material requirements is the specification for light, loose riprap material (Table 9).

Table 9
WSDOT Light Loose Riprap Gradation Specification

Size Range	Maximum Size
20% to 90%	300 lbs to 1 ton (2 cubic feet to 1/2 cubic yards)
15% to 80%	50 lbs to 1 ton (1/3 cubic feet to 1/2 cubic yards)
10% to 20%	50 lbs (spalls)

4.7 Filter Material Calculations

It is recommended that a filter layer material be placed between the regraded shoreline slope and the new shoreline armor material to prevent migration of fine soil particles, distribute the weight of the armor units, provide more uniform settlement, and permit relief of hydrostatic pressure within the soils (USACE 1995).

Using guidance presented in Engineering Manual (EM) 1110-2-1614 (USACE 1995), the selected filter layer must satisfy requirements pertaining to both the armor-to-filter relation as well as filter-to-underlying soil relation as defined by the following three equations:

$$\frac{\mathrm{D}_{15(\mathrm{armor})}}{\mathrm{D}_{85(\mathrm{filter})}} < 4 \qquad \text{(Equation 1)}$$

$$\frac{D_{15(\mathrm{filter})}}{D_{85(\mathrm{soil})}} < 4\ to\ 5 \qquad \text{(Equation 2)}$$

$$4 \text{ to } 5 < \frac{\mathrm{D}_{15(\mathrm{filter})}}{\mathrm{D}_{15(\mathrm{soil})}} \qquad \text{(Equation 3)}$$

Equation 1 provides a margin against variations in void sizes that may occur as the armor layer shifts under wave action. Equation 2 is intended to prevent vertical migration of the underlying soil through the filter (often referred to as "piping"). Equation 3 provides for adequate permeability for structural bedding layers (USACE 1995).

Table 10 presents the material gradation specification for the proposed filter layer material.

Table 10
Filter Material Gradation Specification

U.S. Standard Sieve Size	Percent Passing by Weight
4 inch	90 - 100
3/4 inch	50 - 75
No. 4	35 - 55
No. 10	25 - 45
No. 40	10 – 25
No. 200	0 – 4 (wet sieve)

Figures 1, 2, and 3 depicts the filter layer requirements based on Equations 1 through 3 and the proposed filter layer material gradation specification outlined in Table 10. As seen on the figures, the range for D_{85} in the filter material specification meets or exceeds the required range based on the relationship to the D_{15} of the armor layer material specification (Equation 1). The range for D_{15} in the filter material specification exceeds the range based on the relationship to the underlying soil (Equation 2). Generally, the D_{15} in the filter material specification meets the range for permeability based on the underlying soil (Equation 3), with the exception of locations coarser-grained sediment is present.

5 CONCLUSIONS

Based on the analysis of the erosive forces expected at the RAB (vessel-generated wakes, propeller wash, river currents), the following protective material is recommended for the backfill and shoreline armoring:

<u>Backfill</u>

- The controlling long-term erosive force on the backfill is river currents.
- To protect against river current material with a D_{50} of 6 mm should be used at least in the upper foot of backfill, if not throughout.
- This backfill grain size should address temporary transient propeller wash forces.

<u>Shoreline Armoring</u>

The 100-year river current event is the dominate erosive force expected within the RAB. A rock gradation with a D_{50} of 0.9 foot is recommended to provide the appropriate protection the full height of the 2H:1V slope.

The use of a filter layer is recommended to prevent migration of fine soil particles, to distribute the weight of the armor units, to provide more uniform settlement, and to permit relief of hydrostatic pressure within the soils.

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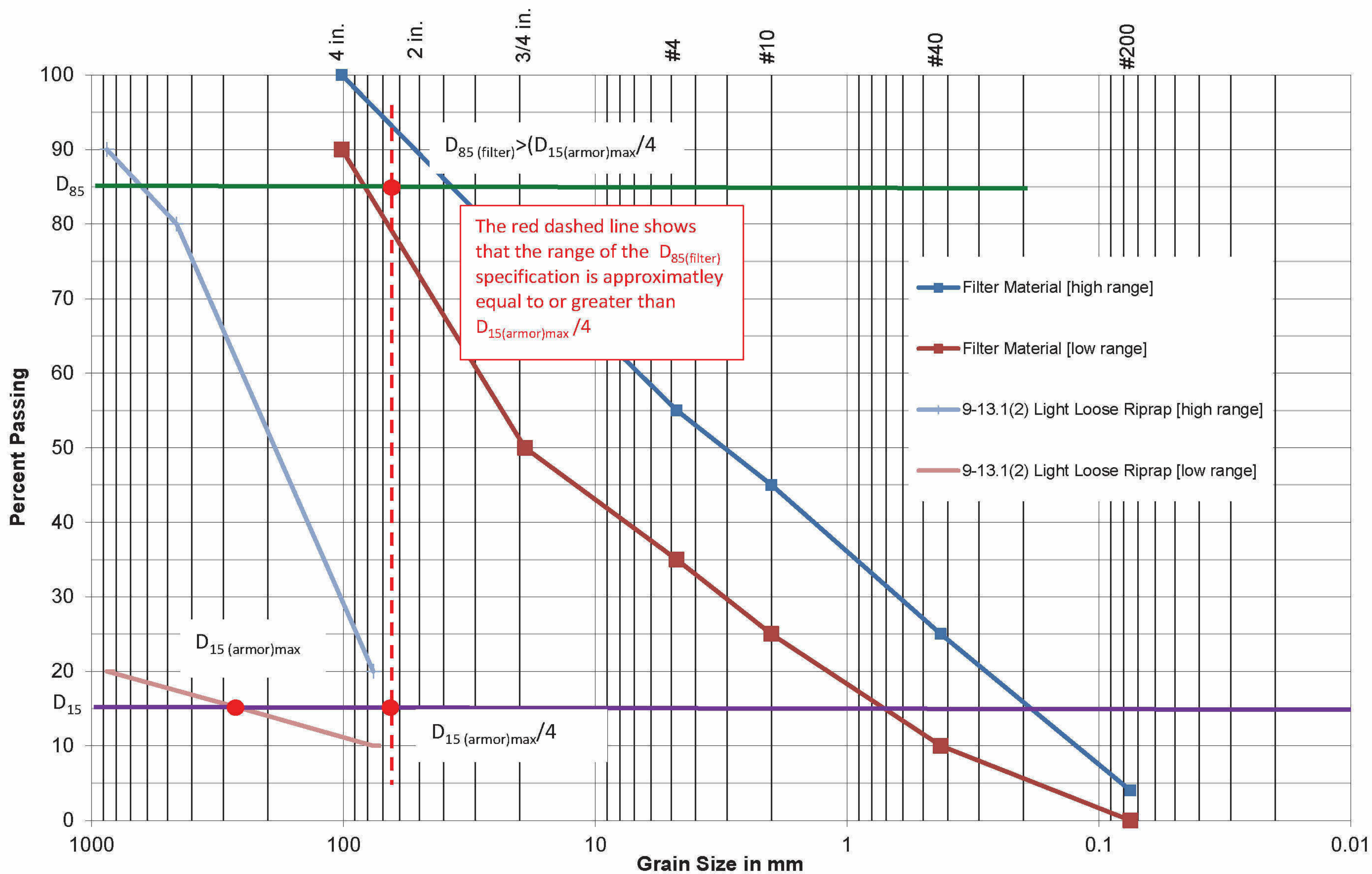
FIGURES



ANCHOR QEA

Figure 1
Filter Material Design Criteria for Void Variation Sizes
Erosion Analysis
Jorgensen Forge Early Action Area

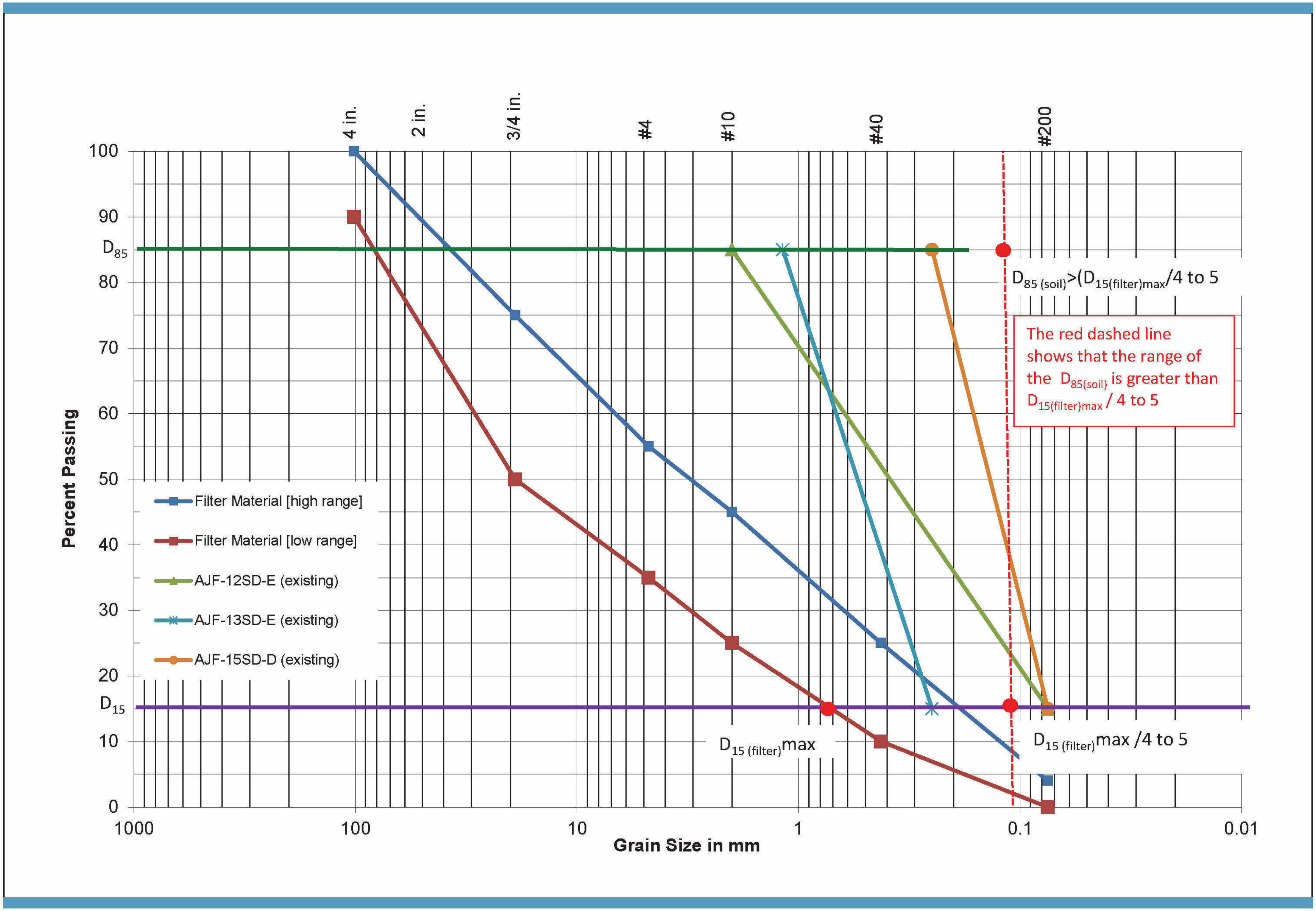


Figure 2
Filter Material Design Criteria for Vertical Migration
Erosion Analysis
Jorgensen Forge Early Action Area

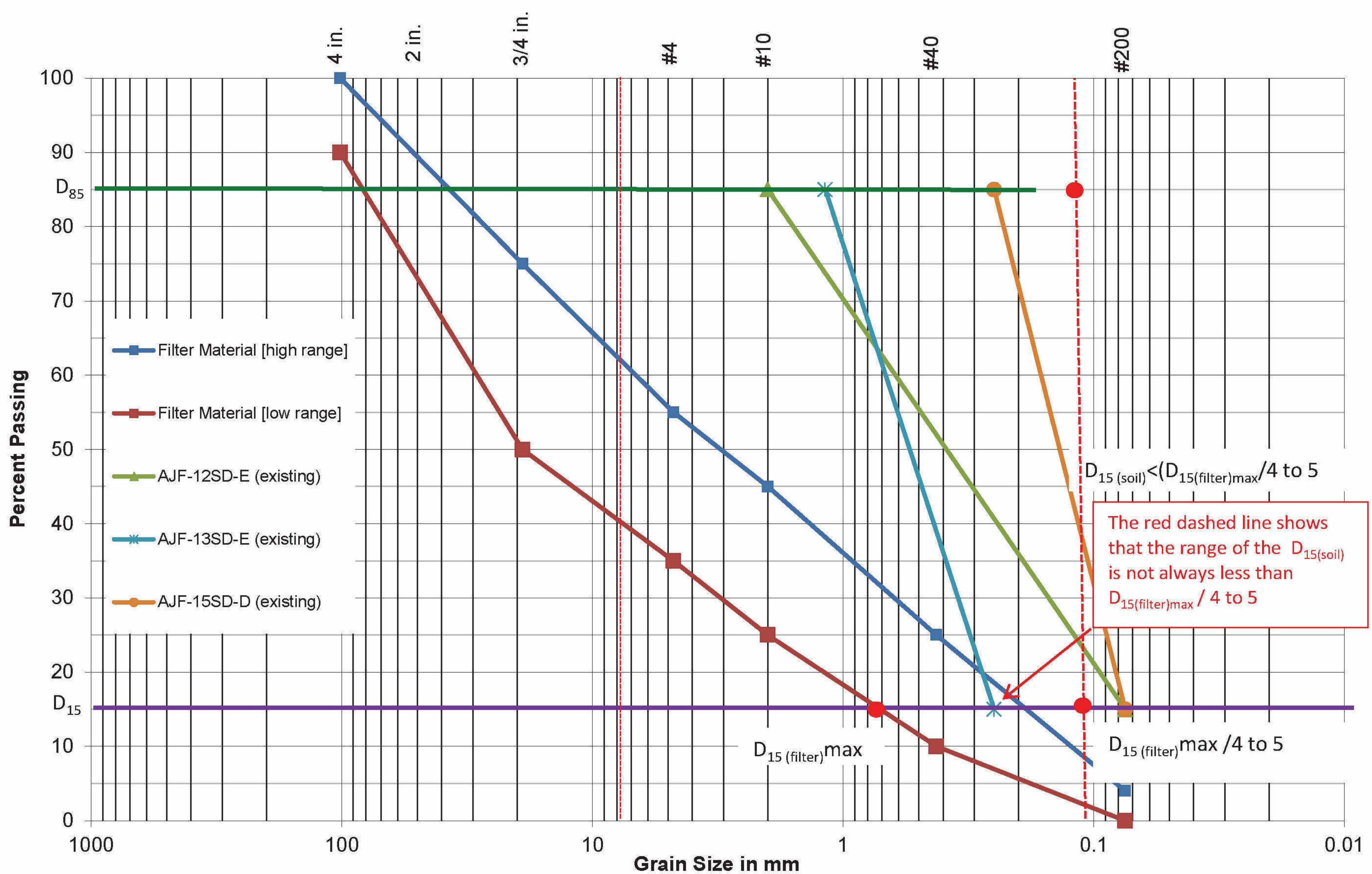


Figure 3
Filter Material Design Criteria for Permeability
Erosion Analysis
Jorgensen Forge Early Action Area

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ATTACHMENT 1
VESSEL WAKE ANALYSIS FOR ARMOR LAYER DESIGNS – EXAMPLE CALCULATION

CALCULATION COVER SHEET

PROJECT: Jorgensen Forge	CALC NO. 1	SHEET 1 of 6
SUBJECT: Attachment 1 – Vessel Wake Analysis for Armor Layer Designs - Example Calculation		

Objective: To determine the wave height and period generated by a vessel traveling through the Jorgensen Forge Project site

References:

Windward and QEA, 2008. Final Lower Duwamish Waterway Sediment Transport Analysis Report. Prepared for the Lower Duwamish Waterway Group. January 24.

AMEC Floyd Snider, 2011. Appendix I – Vessel Propeller Wash And Wake Scour Analysis. Prepared for The Boeing Company.

Weggel, J.R. and R.M. Sorensen. 1986. "Ship wave prediction for port and channel design." Proceedings of the Ports '86 Conference, Oakland, CA, May 19-21, 1986. Paul H. Sorensen, ed., American Society of Civil Engineers, New York, pp. 797-814.

Determination of wake wave height and period for a tugboat: The following presents a detailed summary and example calculation to determine the wave height and period of a wake wave generated by a tugboat traversing the Duwamish River. The approach was developed by Weggel and Sorensen (1986) and Sorensen and Weggel (1984). The numbered list below outlines the general approach used for the calculation and defines specific parameters used in the calculations.

1. Obtain vessel characteristics (model input parameters) for the vessel in question, in this case the *Patricia S*, a tugboat operated by Island Tug and Barge. Also, determine water depth and distance to sailing line, where wave characteristics will be assessed. These parameters are provided in the following table:

Table A-1
Vessel Characteristics and Input Parameters (Tugboat)

Parameter	Value	Units
Length	93	feet
Vessel Displacement	20,700	cubic feet
Vessel Speed	8.1	mph
Water Depth	15	feet

2. Relating maximum wave height, Hm, to the vessel speed, distance from the sailing line, water depth, and the vessel displacement yields four dimensionless variables (equations 1 through 4) with their corresponding values for this calculation:

$$F = \frac{V}{\sqrt{gd}}$$

$$x^* = \frac{x}{W^{0.33}}$$

$$d^* = \frac{d}{W^{0.33}}$$

$$H_m^* = \frac{H_m}{W^{0.33}}$$

Where
F = Froude number
V = vessel speed
g = acceleration of gravity
d* = dimensionless water depth
d = water depth
x* = dimensionless distance from vessel sailing line to point of interest
x = distance from vessel sailing line to point of interest measured perpendicular to the sailing line
W = vessel displacement
H_m^* = dimensionless maximum wave height
H_m = maximum wave height in a vessel wave record

3. The basic initial model, in terms of these dimensionless variables, is given by (equation 5):

$$H_m^* = \alpha(x^*)^n$$

Where α and *n* are a function of the Froude number and dimensionless depth as follows (equation 6):

$$n = \beta(d^*)^\delta$$

Where (equation 7):

$$\beta = -0.342 \qquad 0.55 < F < 0.8$$
$$\beta = -0.225\, F^{-0.699} \qquad 0.2 < F < 0.55$$

$$\delta = -0.146 \qquad 0.55 < F < 0.8$$
$$\delta = -0.118\, F^{-0.356} \qquad 0.2 < F < 0.55$$

and (equation 8):

$$\log(\alpha) = a + b \log(d^*) + c \log^2(d^*)$$

Where (equation 9):

$$a = \frac{-0.6}{F}$$

$$b = 0.75F^{-1.125}$$

$$c = 2.653F - 1.95$$

Where:
a,b, c, β, and δ = Dimensionless coefficients

4. Using Equations 5 through 9, H_m can be determined given the vessel speed, displacement, water depth, and distance from the sailing line. These equations are valid for vessel Froude numbers from 0.2 to 0.8, which are common for most vessel operations, and in this case is 0.54 as defined in equation 1 above (and shown in the calculation below).

$$F = \frac{V}{\sqrt{gd}} = \frac{8.1\,\frac{\text{miles}}{\text{hr}} \times 5{,}280\,\frac{\text{ft}}{\text{mile}} \times \frac{1}{3{,}600}\,\frac{\text{hr}}{\text{sec}}}{\sqrt{32.2\,\frac{\text{ft}}{\text{s}^2} \times 15\text{ ft}}} = 0.54$$

Where:
V = 8.1 miles per hour
g = 32.2 ft/s^2
d = 15 feet

Given F = 0.54, □ = -0.35 and □ = -0.15 and the value of Hm = 0.5 ft

equation 2:

$$x^* = \frac{x}{W^{1/3}} = \frac{100\text{ ft}}{\left(20{,}700\text{ ft}^3\right)^{1/3}} = 3.64$$

equation 3:

$$d^* = \frac{d}{W^{1/3}} = \frac{15\text{ ft}}{\left(20{,}700\text{ ft}^3\right)^{1/3}} = 0.55$$

equation 4:

$$H_m{}^* = \frac{H_m}{W^{1/3}} \Rightarrow H_m = \left(H_m{}^*\right)\left(W^{1/3}\right) = 0.02 \times \left(20{,}700\text{ ft}^3\right)^{1/3} = 0.5\text{ ft}$$

equation 5:

$$H_m{}^* = \alpha(x^*)^n = 0.03 \times (3.64)^{-0.3} = 0.02$$

equation 6:

$$n = \beta (d^*)^{\delta} = -0.35 \times (0.55)^{-0.146} = -0.38$$

Equation 8:

$$\log(\alpha) = a + b\log(d^*) + c\log^2(d^*) = -1.1 + 1.5 * \log(0.55) + -0.52 * \log^2(0.55) = -1.52$$

$$\alpha = 10^{-1.52} = 0.03$$

Equation 9:

$$a = \frac{-0.6}{F} = \frac{-0.6}{0.54} = -1.1$$

$$b = 0.75F^{-1.125} = 0.75(0.54)^{-1.125} = 1.5$$

$$c = 2.653F - 1.95 = 2.653 \times 0.54 - 1.95 = -0.52$$

Where:
F = 0.54 (per equation 1 above)
V = miles per hour
g = 32.2 ft/s^2
d = 15 feet
x = = 100 feet
W = 20,700 ft^3

5. The wave height is subsequently adjusted by modifying the value of H_m by the following relationship (equation 10):

$$H_m = A'H_m - B' = 3.30 \times 0.5\,\text{ft} - 0.145 = 1.5\,\text{ft}$$

Where,
A' and B' = coefficients to account for hull geometry = 3.30 and 0.145 (Table 2 of Weggel and Sorensen 1986; Kurata & Oda[1984] tugboat)

6. In order to determine the wave period, the diverging wave direction is determined with respect to the sailing line, by the following equation (equation 15):

$$\theta = 35.27 - 35.27e^{(12F-12)} \qquad F<1$$

$$\theta = a\sin\left(\frac{1}{F}\right) \qquad F>1$$

In this example calculation where F= 0.54:

$$\theta = 35.27 - 35.27e^{(12*0.54-12)} = 35.13 \text{ degrees, or } 0.61 \text{ radians}$$

And the diverging wave celerity, C is determined by the following (equation 16):

$$C = V\cos(\theta) = 8.1\frac{\text{miles}}{\text{hr}} \times 5{,}280\frac{\text{ft}}{\text{mile}} \times \frac{1}{3{,}600}\frac{\text{hr}}{\text{sec}} \times \cos(35.13) = 9.7\frac{ft}{\text{sec}}$$

Where
V = 8.1 mph

And the period is calculated as (equation 17):

$$T = 2\pi(C/g) \qquad F<0.7$$

$$T = \frac{L^*}{C} \qquad F>0.7$$

Where
L* is determined through an iterative process, to equate C with C^*, where C* is defined as (equation 18):

$$C^* = \sqrt{\frac{gL^*}{2\pi}\tanh\left(\frac{2\pi d}{L^*}\right)}$$

In this example F < 0.7, and the first part of equation 17 is used to determine T:

$$T = 2\pi\left(\frac{9.7\frac{ft}{\text{sec}}}{32.2\frac{ft}{\text{sec}^2}}\right) = 1.9\,\text{sec}$$

7. Compute the Armor Stone Size Along the Shoreline

The *Rubble Mound Revetment Design Module* in ACES was used to compute the required armor layer size (gradation and thickness) in the surf zone to resist the forces generated by turbulence from breaking waves. The following parameters were used in the computation:

- Significant wave height = 1.5 feet
- Significant wave period = 2.0 seconds
- Breaking criteria = 0.78 (Dean and Dalrymple 1991)
- Water depth at toe of the structure = 16 feet

- Cotangent of nearshore slope = 6 (the slope of the bed offshore of the surf zone in Remediation Area)
- Unit weight of rock = 165 lbs/ft^3 (page A-6 of Maynord 1998)
- Permeability coefficient = 0.4 (Figure 4-4-2b of USACE 1992)
- Cotangent of structure (revetment) slope = 2 (restored slope for Remediation Area)
- Minor Displacement Level (S) = 2 (from Table VI-5-21 of USACE 2006 and Table 4-4-1 of USACE 1992)

Table A-2 presents the armor layer gradation results for the minor displacement level for a 2H:1V slope computed by ACES.

Table A-2

Shoreline Armor Gradation for Minor Displacement for Remediation Area

Gradation and Thickness	Stone Size (inches) for Minor Displacement (S=2)
D_0	2.4
D_{15}	3.6
D_{50}	4.8
D_{85}	6.0
D_{100}	7.2
Thickness of Armor Layer	9.6

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ATTACHMENT 2
PROPELLER WASH ANALYSIS FOR SEDIMENT BACKFILL LAYER DESIGN

CALCULATION COVER SHEET

PROJECT: Jorgensen Forge	CALC NO. 1	SHEET 1 of 3
SUBJECT: Attachment 2 – Propeller Wash Analysis for Sediment Backfill Layer Design		

Objective: To determine the propeller wash velocities from commercial and recreational vessels that may operate at the Jorgensen Forge project site and the resultant particle size(s) necessary for stability of the sediment cap subject to these propeller wash flows.

This document presents the calculations for the Island Tug and Barge Company Tug Tugboat *Patricia S.*

References:

Blaauw, H.G., and E.J. van de Kaa. 1978. "Erosion of Bottom and Sloping Banks Caused by the Screw Race of Maneuvering Ships." Paper presented at the 7th International Harbour Congress, Antwerp, Belgium. May 22-26, 1978.

Maynord, S. 1998. *Appendix A: Armor Layer Design for the Guidance for In-Situ Subaqueous Capping of Contaminated Sediment.* Prepared for the U.S. Environmental Protection Agency (USEPA).

Weggel, J.R. and R.M. Sorensen. 1986. "Ship wave prediction for port and channel design." Proceedings of the Ports '86 Conference, Oakland, CA, May 19-21, 1986. Paul H. Sorensen, ed., American Society of Civil Engineers, New York, pp. 797-814.

Windward and QEA, 2008. Final Lower Duwamish Waterway Sediment Transport Analysis Report. Prepared for the Lower Duwamish Waterway Group. January 24.

AMEC Floyd Snider, 2011. Appendix I – Vessel Propeller Wash And Wake Scour Analysis. Prepared for The Boeing Company.

Computation of commercial vessel propeller wash and resultant particle size(s): The following presents a detailed example calculation for a commercial vessel operating on the Duwamish River. The numbered list below outlines the general approach used for the calculation and defines specific parameters used in the calculations. Subsequent sections below illustrate a step-by-step calculation for the example case. The calculation is for the *Patricia S* tugboat operating in 15 ft of water (low tide) at 25 percent of the installed engine power.

1. Select representative vessel for analysis

The *Patricia S* tugboat was the example vessel used in the calculation to represent tugboats operating on the Lake. The tugboat has the following characteristics:

- Number of engines: Two
- Propeller shaft depth: 8 feet (ft)
- Total installed engine horsepower: 2400 horsepower (hp)
- Propeller diameter: 6.3 ft
- Ducted propeller: Yes

2. Determine the maximum bottom velocities in the propeller wash of a maneuvering vessel

Equation 4 from Maynord (1998) is used to first determine the jet velocity exiting a propeller (U_0) in feet per second (fps):

$$U_0 = C_2 \left(\frac{P_d}{D_p^{\ 2}} \right)^{\frac{1}{3}}$$

Where:
C_2 = 7.68 for ducted propellers (page A-10 from Maynord 1998)
P_d = applied engine horsepower = 300 hp
D_p = Propeller diameter = 6.3 ft (from above)

In this analysis it is assumed an average of 25 percent of the engines horsepower is applied, i.e. P_d = 0.25x1200 hp = 300 hp. Therefore,

$$U_0 = C_2 \left(\frac{P_d}{D_p^{\ 2}} \right)^{\frac{1}{3}} = (7.68)\left(\frac{300}{6.3^2} \right)^{\frac{1}{3}} = 15.1\,\text{fps}$$

The resulting maximum bottom velocities, $V_{b(maximum)}$, in the propeller wash of a maneuvering vessel is computed using Equation 3 from Maynord (1998):

$$V_{b(maximum)} = C_1 U_0 D_p / H_p$$

Where:
C_1 = 0.30 for a ducted propeller
H_p = distance from propeller shaft to channel bottom in ft

In this calculation, the *Patricia S* is operating in a depth of 15 feet of water (low water). Therefore, the distance form the propeller shaft to channel bottom is the water depth minus the shaft depth (i.e., H_p = 15 ft- 8 ft = 7 ft). The maximum bottom velocity for this case is:

$$V_{b(maximum)} = C_1 U_0 D_p / H_p = 0.30(15.1)(6.3)/7 = 4.1 \text{ fps}$$

3. Compute the Stable Sediment Sizes to resist the propeller wash of a maneuvering vessel

Equation 5 from Maynord (1998) is used to compute the Stable Sediment Sizes to resist the propeller wash of a maneuvering vessel:

$$V_{b(\mathrm{max}imum)} = C_3\left[g\left(\frac{\gamma_s - \gamma_w}{\gamma_w}\right)D_{50}\right]^{\frac{1}{2}}$$

Where:

C_3 = 0.6 for infrequent attacked expected (page A-10 from Maynord 1998)
D_{50} = median particle size
γ_s = unit weight of stone = 165 pounds per cubic foot (lbs/ft^3) (page A-6 of Maynord 1998)
γ_w = unit weight of sea water = 64.0 lbs/ft^3

Solving for D_{50}:

$$D_{50} = \frac{\left(\frac{4.1}{0.6}\right)^2}{32.2\left(\frac{165-64.0}{64.0}\right)} = 0.91\,\mathrm{ft} = 10.9 \text{ inches}$$

It should be noted that this method provides a conservative estimate of stable particle size for the low bottom velocities when compared with other methods used to compute a representative particle size to resist erosion associated with current velocities. For example, the stable particle size to resist a 4.1 fps bottom current velocity using Shields diagram presented in Vanoni (1975) is 0.8 inches (21 millimeters).

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ATTACHMENT 3
CURRENT VELOCITY AND ARMOR SIZE CALCULATIONS

CALCULATION COVER SHEET

PROJECT: Jorgensen Forge	CALC NO. 1	SHEET 1 of 5
SUBJECT: Attachment 3 – Current Velocity and Armor Size Calculations		

Objective: To determine current velocities present adjacent to the site for a 100-year return period event and use these velocities to calculate an appropriate armor stone size for the embankment.

References:

Dingman, S.L. Fluvial Hydraulics. Oxford University Press. February 26, 2009.

Federal Emergency Management Agency (FEMA). Flood Insurance Study, King County, WA and incorporated areas. FEMA. April 19, 2005.

Palermo, M., Maynord, S., Miller, J., and Reible, D. Guidance for In-Situ Subaqueous Capping of Contaminated Sediments, EPA 905-B96-004. EPA Great Lakes National Program Office, Chicago, IL. 1998

Windward and QEA, 2008. Final Lower Duwamish Waterway Sediment Transport Analysis Report. Prepared for the Lower Duwamish Waterway Group. January 24.

QEA, 2008. Lower Duwamish Waterway Sediment Transport Modeling Report – Final. Prepared for the U.S. Environmental Protection Agency and Washington State Department of Ecology. October.

Verification of bottom velocity data. To ensure that the velocities used to determine the armor stone size are of sufficient magnitude to represent extreme conditions at the site, the velocities shown in STM Figure E-18 (QEA 2008) were supplemented with velocities back-calculated from the maximum bottom shear stresses at the site. This was accomplished using Equations A-2 through A-4 in Appendix A of the STM, reiterated and detailed below:

$$\tau = \rho_w C_f u^2$$

$$C_f = \kappa^2 \ln^{-2}\left(11\frac{z_{ref}}{k_s}\right)$$

$$z_{ref} = 0.5\frac{\eta - h}{10}$$

$$k_s = 2D_{90}$$

Where:
τ = bottom shear stress due to skin friction (Pa)
ρ_w = density of water (1000 kg/m^3)
C_f = skin friction coefficient
u = near-bottom current velocity (m/s)
κ = Von Karman's constant (0.4)
z_{ref} = reference height above bed (m)
k_s = effective bed roughness height (m)
η = water surface elevation (m MSL)
h = bottom elevation (m MSL)
D_{90} = 90 percentile grain diameter (m)

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Table B-6 of the STM shows that the D_{90} of the cohesive sediment on the eastern bench near the site is 940 microns. The effective bed roughness is then:

$$k_s = 2D_{90} = 2 * 0.00094 = 0.00188m$$

In the case of the largest modeled shear stress (4.0 Pa), the lowest bottom elevation is -25 ft MSL and the highest water surface elevation is +6 ft MSL. The reference height is then calculated:

$$z_{ref} = 0.5\frac{\eta - h}{10} = 0.5\frac{6 - -25}{10} = 1.55\text{ft} = 0.47\text{m}$$

The coefficient of skin friction is then:

$$C_f = \kappa^2 \ln^{-2}\left(11\frac{z_{ref}}{k_s}\right) = 0.4^2 \ln^{-2}\left(11\frac{0.47}{0.00188}\right) = 0.0026$$

Finally, the velocity can be calculated:

$$u = \sqrt{\tau/\rho_w C_f} = \sqrt{4.0/1000(0.0026)} = 1.25\text{m/s} = 4.1\text{ft/s}$$

Calculating depth-averaged velocity from bottom shear data. For sizing armor along the banks, a depth-averaged velocity over the entire water column is more appropriate. The depth-averaged velocity can be estimated using the 'law of the wall', in which an idealized logarithmic velocity is developed from the bottom shear stress and the flow and sediment characteristics (Dingman 2009). The equations below show this method in detail as it is applied here.

$$u = \frac{u_*}{\kappa}\ln\left(y/y_0\right)$$

$$u_* = \sqrt{\tau/\rho_w}$$

$$y_0 = k_s/30$$

$$k_s = 2D_{90}$$

Where:
u = horizontal velocity as a function of height from bottom (m/s)
u_* = shear velocity (m/s)
κ = Von Karman's constant (0.4)

y = height from bottom of channel (m)
y_0 = reference height for rough turbulent flow (m)
τ = bottom shear stress (Pa, from Table 3)
ρ_w = density of water (1000 kg/m^3)
k_s = effective bed roughness height (m)
D_{90} = 90 percentile grain diameter (m)

Using the same depth and bottom shear scenario as above, the calculations are as follows. The effective bed roughness is first determined:

$$k_s = 2D_{90} = 2 * 0.00094 = 0.00188m$$

The reference height is based on the effective bed roughness:

$$y_0 = {k_s}/{30} = {0.00188}/{30} = 6.27 * 10^{-5}\text{m}$$

The shear velocity is based on bottom shear stress and water density:

$$u_* = \sqrt{{\tau}/{\rho_w}} = \sqrt{{4.0}/{1000}} = 0.0632\text{m/s}$$

The velocity profile over the entire water column is then defined as:

$$u = \frac{u_*}{\kappa}\ln\left({y}/{y_0}\right) = 0.1581\ln\left({y}/{6.27 * 10^{-5}}\right)$$

Values of velocity were calculated on a fine interval over the entire 31 ft (9.45 m) water column and the average value was found to be 173 cm/s (5.7 ft/s).

Determining the required armor stone size based on velocity and bank slope. The Maynord formulation (Appendix A, Palermo, 1998) was used to estimate a median riprap diameter and weight based on the largest depth-averaged velocity in Table 4 of 5.7 ft/s. The formula is detailed below.

$$W_{50} = \gamma_s {D_{50}}^3$$

$$D_{50} = S_f C_s C_v C_t C_g d\left[\sqrt{\frac{\gamma_w}{\gamma_s - \gamma_w}}\frac{V}{\sqrt{K_1 g d}}\right]^{2.5}$$

$$K_1 = \sqrt{1 - \frac{\sin^2 \theta}{\sin^2 \varphi}}$$

$$C_v = 1.283 - 0.2 \log\left({}^{R}/_{W}\right)$$

$$C_g = \left(\frac{D_{85}}{D_{15}}\right)^{1/3}$$

where:
W_{50} = median stone weight (lb)
D_{50} = median stone diameter (ft)
γ_s = unit weight of stone (165 lb/ft^3)
γ_w = unit weight of water (62.4 lb/ft^3)
S_f = factor of safety (2.0)
C_s = stability coefficient (0.30 for angular stone)
C_v = velocity distribution coefficient
C_t = blanket thickness coefficient (1.0 for flood flows)
C_g = gradation coefficient
V = depth-averaged velocity (ft/s)
g = gravitational acceleration (32.2 ft/s^2)
d = water depth (ft)
K_1 = side slope correction factor
Θ = revetment slope ($\tan^{-1}$(rise/run))
Φ = riprap angle of repose (40°)
R = radius of curvature of the bend (2,300 ft)
W = surface width upstream of the bend (400 ft)
D_{85}/D_{15} = ratio of 85 percentile armor size to 15th percentile armor size (4.0)

This example uses the maximum depth-averaged velocity expected in the area (5.7 ft/s) and a bank slope of 1V:2H (26.6 degrees). First, the gradation coefficient is calculated:

$$C_g = \left(\frac{D_{85}}{D_{15}}\right)^{1/3} = (4.0)^{1/3} = 1.59$$

The velocity distribution coefficient is based on the width of the river and the radius of the bend:

$$C_v = 1.283 - 0.2 \log\left({}^{R}/_{W}\right) = 1.283 - 0.2 \log\left({}^{2300}/_{400}\right)$$
$$= 1.131$$

The side slope correction factor is based on the slope of the bank and the riprap angle of repose (40 degrees):

$$K_1 = \sqrt{1 - \frac{\sin^2 \theta}{\sin^2 \varphi}} = \sqrt{1 - \frac{\sin^2 26.6}{\sin^2 40}} = 0.718$$

The median armor stone diameter is then calculated using the Maynord equation for a depth of 1.0 ft:

$$D_{50} = S_f C_s C_v C_t C_g d \left[\sqrt{\frac{\gamma_w}{\gamma_s - \gamma_w}} \frac{V}{\sqrt{K_1 g d}} \right]^{2.5}$$

$$= 2.0 * 0.30 * 1.131 * 1.0 * 1.59 * 1.0 \left[\sqrt{\frac{62.4}{165 - 62.4}} \frac{5.7}{\sqrt{0.718 * 32.2 * 1.0}} \right]^{2.5}$$

$$= 0.88\text{ft}$$

Finally, the median armor weight is found by assuming the median diameter represents the 'equivalent cube side length' of the stone:

$$W_{50} = \gamma_s {D_{50}}^3 = 165 * 0.88^3 = 114 lb$$

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